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Numerical assessment of the influence of different joint hysteretic models over the seismic behaviour of Moment Resisting Steel Frames

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Abstract. The main aim of this work is to understand how the prediction of the seismic performance of moment-resisting (MR) steel frames depends on the modelling of their dissipative zones when the structure geometry (number of stories and bays) and seismic excitation source vary. In particular, a parametric analysis involving 4 frames was carried out, and, for each one, the full-strength beam-to-column connections were modelled according to 4 numerical approaches with different degrees of sophistication (Smooth Hysteretic Model, Bouc-Wen, Hysteretic and simple Elastic-Plastic models). Subsequently, Incremental Dynamic Analyses (IDA) were performed by considering two different earthquakes (Spitak and Kobe). The preliminary results collected so far pointed out that the influence of the joint modelling on the overall frame response is negligible up to interstorey drift ratio values equal to those conservatively assumed by the codes to define conventional collapse (0.03 rad). Conversely, if more realistic ultimate interstorey drift values are considered for the q-factor evaluation, the influence of joint modelling can be significant, and thus may require accurate modelling of its cyclic behavior.

1. Introduction

When steel structures are subjected to increasing-intensity load reversals, they exhibit a significant and complex damage process in its components (low cycle fatigue), involving stiffness and strength deterioration phenomena and pinching. In the end, this process may become responsible of building collapse. In the case of full-strength beam-to-column joints, studied in this work, these phenomena occur at the ends of the connected beam, which normally represents the structural element designed to dissipate the highest amount of the earthquake energy demand.To perform a reliable and accurate study of the actual joint behavior, numerical modelling must be carried out. In this respect, different methods may be adopted:

- The 3D Finite Element Method has the advantage to shed light on important local effects, e.g. prying and contact forces between the bolt and the connection components, and give the possibility of generating extensive parametric studies. In contrast, however, it has the drawback of requiring a remarkable computational effort, as well as the definition of geometric and material nonlinearities of all the elementary parts constituting the joint.

- In the distributed plasticity approach, the constitutive relationship at element level is evaluated by integrating the stress-strain relationship over the cross-section. This method allows for accurate analyses with reduced computational effort, and is widely employed for modelling the column elements of a steel frame since it is able to capture the variation of the yielding point due to the M-N interaction. However, it cannot predict any degradation phenomena.
- To model the dissipative zones of the beams, the lumped plasticity approach is usually adopted, defining the member with an elastic element equipped at its ends with nonlinear links, where the plastic hinges will develop.

In this respect, a designer can take into account the different deterioration phenomena occurring in steel components by properly defining a hysteresis model (expressed in terms of M-φ curve) for the nonlinear link describing the rotational behavior of the beam-to-column joint. Widespread models are the Smooth Hysteretic Model (SHM) developed by Sivaselvan and Reinhorn [1], the Bouc-Wen model developed by Bouc, then generalized by Wen [2], and subsequently modified by Baber and Noori [3], the Ramberg and Osgood model [4], the Ibarra-Medina-Krawinkler model [5], [6]. The main difficulty in the use of such models is related to the calibration of a large set of parameters which completely define the cyclic M-φ relationship. Many of them may not have explicit physical meaning, and thus specific techniques based on numerical optimization are needed to calibrate their values [7-11]. In the past, some efforts have been devoted to understand the influence of joint modelling on the seismic response prediction of moment-resisting (MR) steel frames at collapse conditions. In [12], [13] and [14], the authors investigated the influence of phenomena such as stiffness and strength deterioration, pinching, low cycle fatigue, hardening and local softening behaviors, by means of extensive sensitivity analyses on the model parameters describing them. The main outcomes attained in these works show that for conventional limit states the simplest elastic-plastic model is adequate to describe the joint behavior, and the post-elastic effects are not to be defined very precisely. Conversely, when the structure is pushed up to collapse, the influence of strength/stiffness deterioration, as well as pinching, becomes more relevant, and thus (i) accurate hysteretic models should be used, and (ii) appropriate calibration of their parameters is required. In [7] the problem of parameter calibration of accurate hysteretic models was dealt with in the framework of multi-objective calibration, by proposing a procedure taking into account monotonic and cyclic response. In this work, the issue of dissipative zone modelling is examined from a point of view slightly different from the cited works [12-16]. Instead of performing sensitivity analyses on the influence of various parameters of the same model, the effect on the global response given by selecting one model rather than another with different degree of accuracy is investigated. Some hysteretic models for steel members, having different M-φ law (bilinear, multilinear, nonlinear) and differently predicting the aforementioned deterioration phenomena, are calibrated onto the results of an ex-perimental program by means of the multi-objective calibration procedure recently developed [7]. Hence, the models simulate the same experimental response, and the difference among them is given by the inherent different mathematical formulations. The calibrated models are then applied to different steel frames to investigate their influence on the global response.

2. Experimental test and parameter calibration

The parameter calibrations herein presented are based on the experimental activities carried out at the Laboratory of Materials and Structures of the University of Salerno, investigating the ultimate behavior of steel beams under cyclic loading conditions [17]. The main purpose of the experimental program was providing findings about the main response parameters which govern the ultimate behavior of steel elements: the rotation capacity and the flexural ultimate resistance. The experimental program dealt with a wide range of cross-sectional shapes (hollow sections, standard open profiles), but in this work IPE 300 only has been considered, since it represents a common profile adopted for beams in the real design practice. Moreover, the tested specimens were designed so that the beam-to-column connection was full-strength, i.e. the beam end undergoes plastic deformations, while the connection remains in elastic range. The testing program included a monotonic (with a loading rate equal to 0.25 mm/s) and a cyclic (according to the AISC 341-05 loading protocol) in-plane bending test on a cantilever beam. In addition, standard tensile coupon tests were performed on specimens extracted from flanges and webs to obtain the material stress–strain curves. The test results provided the following average values for the yielding and ultimate resistance, respectively: fy = 332.25 MPa, fu = 446.25 MPa. The bending test results showed, as expected due to low-cycle fatigue, that under cyclic loading conditions the specimen exhibited a rotation capacity lower than that exhibited under monotonic loading. Conversely, the flexural overstrength under cyclic conditions was similar to that occurred under monotonic loading (Figure 1).

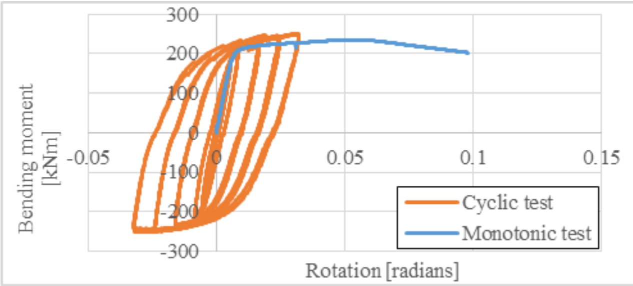


Figure 1. Experimental test results of IPE 300 profile.

The models adopted in this work, and presented below, are implemented in two Structural Analysis software packages: SeismoStruct and OpenSees. Both the software applications are widely used in the scientific community to study the seismic behavior of frame structures. In particular, the model adopted in SeismoStruct is the SHM [1], which is described by a nonlinear M-φ relationship with a smooth transition between the elastic and the plastic zone. This model accounts for stiffness and strength degradation and pinching, while the gap-closing behavior, as described in [1], has been disregarded as non-relevant in the analyzed case of full-strength joints.Conversely, the models adopted in OpenSees are the Bouc-Wen ([2], [3]) and the Hysteretic models. The Bouc-Wen model (BW) presents a mathematical formulation in which several parameters govern the hysteresis loop shape and smoothness, the tangent stiffness and the material deterioration [18]. The hysteretic model (HM), present in the OpenSees material library, is characterized by a piecewise linear behavior, and it includes all the phenomena described for the Polygonal Hysteretic Model [1]. It can be considered as an extension of the bilinear model, which simulates unloading/reloading stiffness degradation, strength degradation and pinching phenomena. The last model adopted, present in both software packages, is the elastic-perfectly plastic (EP) model, and it was selected as it is the simplest model that a designer can adopt in seismic analyses. It represents a basic tool for comparing the results obtained by the other models, because its parameters have a clear physical meaning, and thus they can be readily estimated from experimental and analytical evaluations. Conversely, parameter calibration of SHM, BW and HM was performed according to the multi-objective procedure proposed in [7]. For each numerical model, the set of parameters $\tilde{p}$ is obtained by solving the following optimization problem for NT tests:

$$\tilde{\boldsymbol{p}} = \arg\min_{\boldsymbol{p}\in\boldsymbol{P}}\{\omega_1(\boldsymbol{p}), \dots, \omega_{N_T}(\boldsymbol{p})\} \quad (1)$$

where $\boldsymbol{\omega_i}(\boldsymbol{p}) = \frac{\|y_{exp,i} - y_{c,i}(\boldsymbol{p})\|}{\|y_{exp}\|}$ is the discrepancy function measuring the inconsistency between the experimental and computed quantities for the i-th test, and P the set of all possible parameter combinations. The minimization of ω is accomplished by means of the Non-dominated Sorting Genetic Algorithms II, NSGA-II [19], implemented in the software MultiCal [19]. The vector y_i contains the quantities of the i-th test on which the discrepancy is minimized, which in this application are the bending moment history for the monotonic test, and the hysteretic energy history for the cyclic test. The results of the parameter calibrations in terms of moment-rotation plots are depicted in Figure 2a-f. For the sake of brevity, the dissipated hysteretic energy plots are not reported, since the experimental and numerical curves showed a good matching for all the models adopted. From the results, it is possible to assert that for the SHM the numerical M-φ curve in the monotonic test has a slightly higher peak than the experimental one, which manifests for lower values of the rotation. From the curves of the cyclic test instead, it is possible to perceive a slightly more marked difference. Nevertheless, the attained results are considered more than satisfactory in fitting the experimental curves. As for the BW model the monotonic test provides a more evident difference with respect to the SHM; in fact, the computed curve does not show softening, but rather an almost horizontal post-elastic branch. This aspect is confirmed also by the cyclic test, from where it can be noted that the plastic branch is approximately horizontal, and both strength and stiffness degradations are almost unnoticeable. Finally, the HM, within the limits of a multilinear model, fits very well the experimental curves; in fact, both the hardening and softening branches of the monotonic test are captured, as well as the stiffness degradation of the cyclic test.

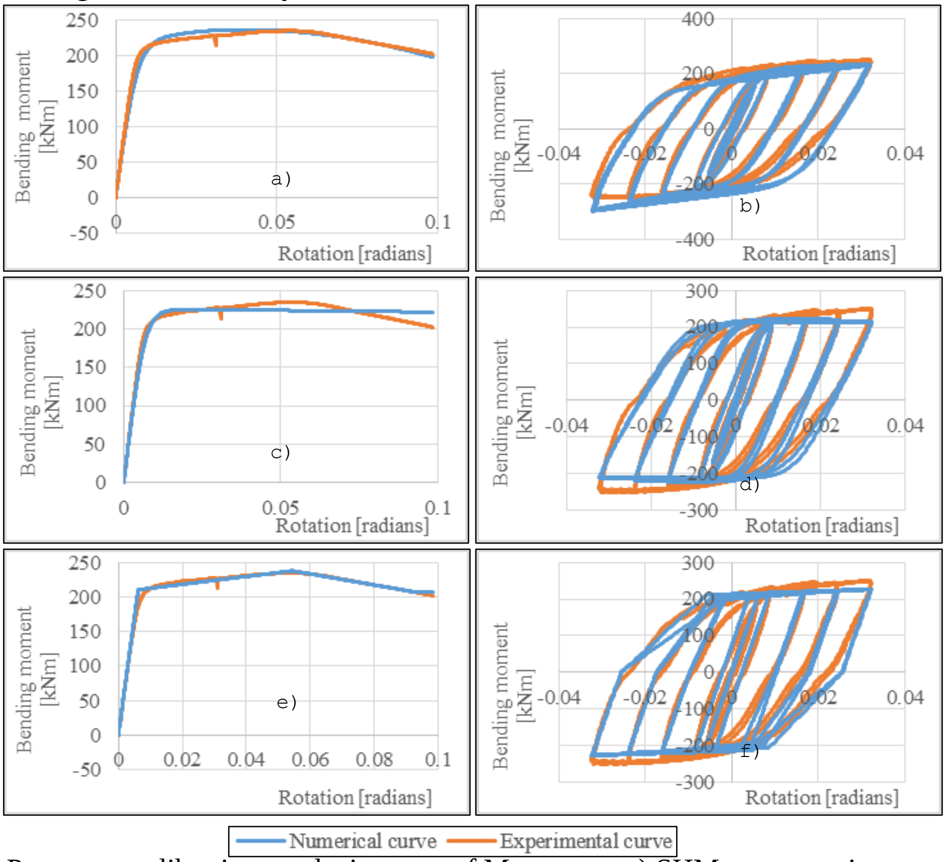


Figure 2. Parameter calibration results in terms of M-φ curve: a) SHM - monotonic test, nb) SHM - cyclic test; c) BW - monotonic test, d) BW - cyclic test; e) HM - monotonic test, f) HM - cyclic test.

3. Parametric analysis

To study the influence of beam-to-column joint modelling on the seismic performance prediction of MR steel frames, four different 2-D frames, starting from a 3 storeys-3 bays frame, up to 6 storeys-5 bays frame, were analyzed. The span length was assumed equal to 6m, while the story height equal to 3.5m for the first floor and 3.2m for the other floors. The frames, extracted from a 3-D structure in which the moment-resistant frames were located at the two extreme sides, were designed and verified according to Eurocode 3 and Eurocode 8 by assuming seismic hazard equal to 0.25g, soil Type C, and finally q-factor value equal to 6.5. As design constraint, constant cross-section IPE 300 was assumed for beams, in order to use the results of the model parameter calibrations. Starting from these assumptions the columns were designed according to the strength hierarchy criterion specified by Eurocode 3 (Table 1).

Table 1. Element cross-sections of the frames analyzed.

Frame	Beams	Columns
3 storeys-3 bays	IPE 300	HEB 300
6 storeys-3 bays	IPE 300	HEM 320
3 storeys-5 bays	IPE 300	HEB 300
6 storeys-5 bays	IPE 300	HEM 360

The numerical modelling of the frames was carried out by assuming a concentrated-plasticity approach for the beams, in which the members were represented by elastic elements with nonlinear zero-length springs at their ends, and a distributed-plasticity representation for the columns. A seismic mass equal to 80t was applied to each floor, and mass-proportional structural damping equal to 5% was assumed. Incremental dynamic analyses (IDA) were performed by considering two ground motion records: Spitak (1988) and Kobe (1995). Newton algorithm was used for the solution of the equations of motion. Each of these cases (4 frame geometrical configurations times 2 ground motion records) has been analyzed by employing all the beam-to-column joint models described in Section 2. Hence, in total 32 analyses were performed.

The Incremental Dynamic Analyses, carried out on the frames by properly scaling the PGA, were stopped when the chord rotation in the first link element modelling the beam-to-column joint achieved the value 0.045 radians, corresponding to the achievement of the ductility of the member observed in the cyclic test [11]. Finally, the q-factor has been computed as the ratio between PGAu at collapse and PGAy at elastic limit.

The IDA curves are shown in Figure 3a-b for two of the cases analyzed. The plots clearly show that the differences between the models in terms of maximum interstorey drift ratio are negligible until the value 0.03 rad, which is generally assumed as collapse state limit in the codes. However, this may not represent the real onset of dynamical instability, and, pushing the analysis forward, the differences between the models become more visible. This, in agreement with the findings of previous research [8-10], implies that it is not necessary to employ very accurate joint modelling as long as the conventional collapse damage level is not exceeded.

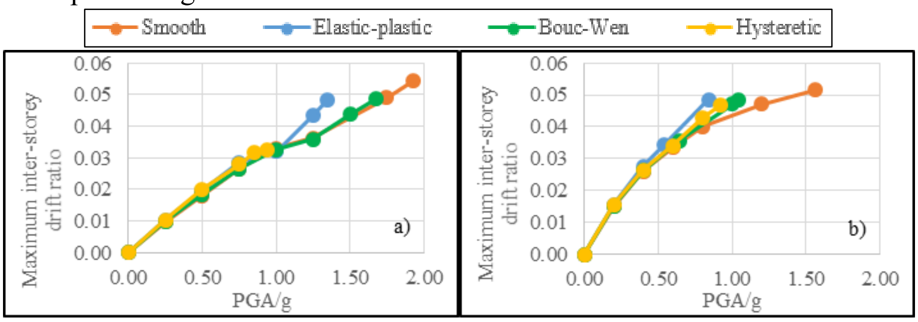


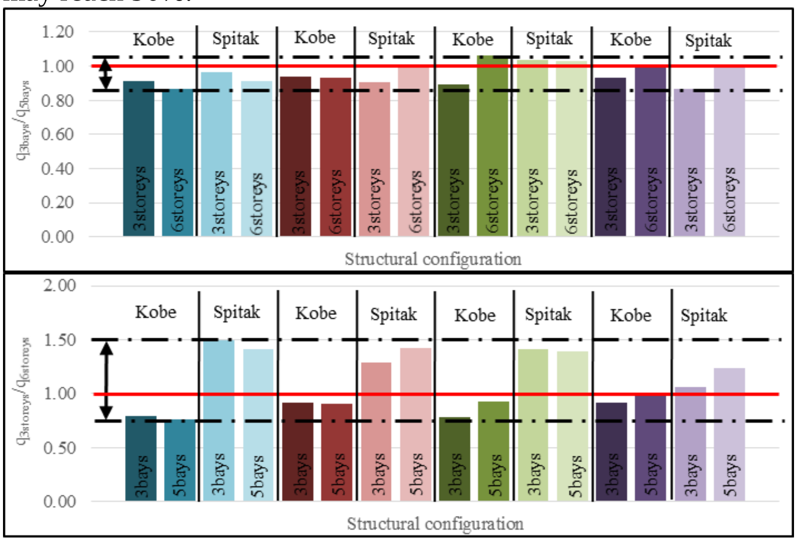
Figure 3. IDA curves in terms of rotation: a) 3 storeys-3 bays frame under Kobe; b) 3 storeys-5 bays frame under Spitak.

To provide a synthetic measure of the influence of joint modelling on the global response of the structure at collapse when varying the parameters (number of bays and stories, ground motion records) investigated in this study, the q-factor was examined. In Figure 4a-c the ratio between the q-factors computed changing one parameter per time is plotted for each structural configuration. In Figure 4a for instance, having fixed the joint model, the earthquake and the number of stories, the ratio between the q-factor evaluated on the 3 bays configuration, and the q-factor evaluated on the same configuration with 5 bays, is computed (q_{3bays}/q_{5bays}). Similarly, the influence of joint modelling has been investigated when the number of stories ($q_{3storeys}/q_{6storeys}$) and the earthquake ($q_{spitak}/q_{kobe}$) changes (depicted respectively in Figure 4b and in Figure 4c). The q-factor ratio represents a measure of the sensitivity of q-factor to the various parameters. If it assumes values close to unity, it means that for

that specific structural configuration the influence of that parameter is low. Extending the view, if this appears in all structural configurations, the conclusion is that the parameter is likely to be noninfluential in general; conversely, any trend observed may suggest more thorough investigations.
The different structural configurations have been grouped on the basis of the same joint model employed, in different color tones: the blue tones refer to the SHM, the red tones refer to the BW, the green tones refer to the HM, and finally the violet tones refer to the EP.

From the analysis results of Figure 4a it is possible to assert that, since the result values fluctuate around 1 in all cases, the effect of the number of bays on the q-factor determination is generally low. Conversely, the influence of the number of stories is evident (Figure 4b); in fact, for almost all cases (model, ground motion record and bay numbers), a large variation in the ratio $q_{3storeys}/q_{6storeys}$ is observed. In particular, a variation of the q-factor ratio between -20% and +50% can be observed for the SHM. This consideration, connected to the previous one, shows that for further parametric analyses it could be more important to investigate the number of floors rather the number of bays.
From Figure 4c another important outcome can be derived; the elastic-plastic model seems not susceptible to the variation of the ground motion record, as in all structural configurations the ratio q_{spitak}/q_{kobe} is close to unity. As the selected ground motions are different in characteristics and frequent content, it is expected that the q-factor differs considerably when evaluated applying any of them, and this occurs for all the models except the elastic-plastic one. For this reason, caution should be used in considering this latter model as reliable in predicting the seismic response over the conventional limits, and this may be a reason to resort to more accurate models.

Finally, in Figure 5, the ratio between the maximum and the minimum q-factor value (q_{max}/q_{min}) searched among the 4 numerical models adopted in the analyses is shown, for each structural configuration and ground motion record. The presence in the histogram of even one single column representing a value larger than 1 means that there is at least one case, among those analyzed, in which the joint modelling has been detected as relevant. Looking at the results, it is evident that for a given structure and ground motion, the evaluation of the q-factor substantially depends on the choice of the numerical model for the hysteretic behavior of the members. For instance, in the case of 6 stories-5 bays frame under Kobe earthquake the difference between q-factor value evaluated considering one model or another may reach 90%.



a

b

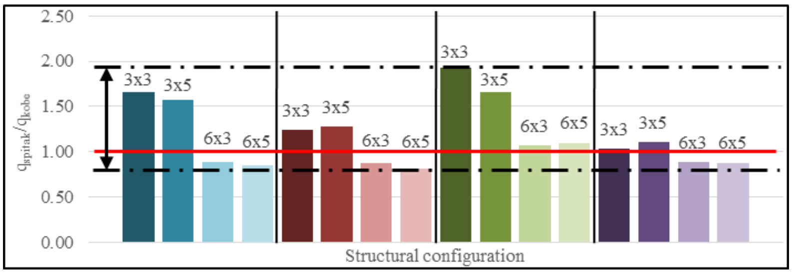


Figure 4. Influence of the various parameters investigated on the seismic response of MR steel frames when varies: a) the number of bays (q_{3bays}/q_{5bays} on the y-axis); b) the number of stories ($q_{3storeys}/q_{6storeys}$ on the y-axis); c) the earthquake type.(q_{spitak}/q_{kobe} on the y-axis).

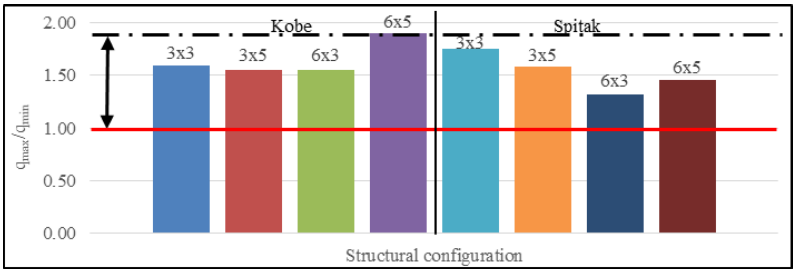


Figure 5. Influence of the joint modelling on the seismic response of MR steel frames.

4. Conclusions

The main purpose of this work was investigating the influence of full-strength beam-to-column joint modelling on the global seismic response of MR steel frames. After preliminary calibration of 4 hysteretic models for steel members with different formulations for the simulation of the post-elastic cyclic behavior, a parametric study consisting of IDA was performed on 32 different structural configurations, in which the numbers of stories and bays, type of earthquake and joint modelling were varied. If the structural response is investigated with reference to the common limit states assumed by the codes (damage LS, life safety LS), i.e. up to a interstorey drift ratio value of about 0.03 rad, a simplified representation as the elastic-plastic model is able to predict results comparable to those of more refined approaches. This conclusion confirms the findings of previous research. Conversely, when larger plastic demand is required to the structure, more evident and important differences can be observed among the numerical models, especially with respect to the behavior factor evaluation. In the cases analyzed, the differences in behavior factor evaluated assuming different modelling for beam-to-column joint reached 90%, meaning that proper modelling of the joint is critical. Furthermore, it has been observed that the global response predicted by the EP model did not appear susceptible to earthquake variations, unlike the other models for which more evident differences were observed. This conclusion appears rather questionable, and thus suggests care in the use of this simplified model for the evaluation of q-factor. Regarding the other parameters analyzed, all numerical models have highlighted low sensitivity of the q-factor to number of bays, and high sensitivity to the number of stories. Even though the analyses performed in this work do not allow for the definition of the most "accurate" model, it seems reasonable to state that a model incorporating all the deterioration phenomena is needed for a realistic estimation of the post-elastic behavior of the structure under earthquake loading up to failure. Looking at the calibration tests, all models seem able to simulate rather well the experimental responses, and so the differences observed in IDAs in terms of response prediction should be verified by means of specific experimental programs. Further extensions of the parametric analyses described herein are planned for future research, directing the efforts to the analysis of the combined influence of model selection for beam-to-column joints and increasing height on the seismic response of buildings, by properly selecting a larger set of ground motion records.

References

[1] Sivaselvan M.V., Reinhorn A.M., Hysteretic models for cyclic behavior of deteriorating in elastic structures, Technical Report MCEER-99-0018, 1999

[2] Ismail M, Ikhouane F, Rodellar J, The hysteresis Bouc-Wen model, a survey, Arch Comput Methods Eng, 16:161–188, 2009

[3] Baber T. T., Noori M. N., Random vibration of degrading pinching systems, J. Eng. Mech., 111(8):1010-1026, 1985

[4] Ramberg W., Osgood W. R., Description of stress-strain curves by three parameters, Technical Note No. 902, National Advisory Committee for Aeronautics, Washington DC, 1943

[5] Ibarra L. F., Medina R. A., Krawinkler H., Hysteretic models that incorporate strength and stiffness deterioration, Earthquake Engineering and Structural Dynamics, 34:1489–1511, 2005

[6] Lignos D. G., Krawinkler H., Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading, Journal of Structural Engineering, 137:1291-1302, 2011

[7] Chisari C., Francavilla A. B., Latour M., Piluso V., Rizzano G., Amadio C., Critical issues in parameter calibration of cyclic models for steel members, Engineering Structures, 132:123–138, 2017

[8] Güneyisi E.M., D'Aniello M., Landolfo R., Mermerdaþ K. (2013) A novel formulation of the flexural overstrength factor for steel beams. Journal of Constructional Steel Research 90 , pp. 60-71

[9] D'Aniello M., Güneyisi E.M., Landolfo R., Mermerdaþ K. (2015) Predictive models of the flexural overstrength factor for steel thin-walled circular hollow section beams. Thin-Walled Structures, 94:67-78. 10.1016/j.tws.2015.03.020

[10] Güneyisi E.M., D'Aniello M., Landolfo R., Mermerdaþ K. (2014) Prediction of the flexural overstrength factor for steel beams using artificial neural network. Steel and Composite Structures, An International Journal, Vol. 17, No. 3, September 2014, pages 215-236

[11] Latour, M., Rizzano, G. Full strength design of column base connections accounting for random material variability (2013) Engineering Structures, 48, pp. 458-471

[12] Landolfo R., Della Corte G., De Matteis G., L'influenza della modellazione dei collegamenti sulla risposta sismica dei telai di acciaio, X Congresso Nazionale "L'ingegneria Sismica in Italia", Potenza-Matera 9-13 Settembre 2001

[13] De Matteis G., Della Corte G., Landolfo R., Influence of connection modelling on seismic response of moment resisting steel frames, Moment Resistant Connections of Steel Frames in Seismic Areas: Design and reliability, pp. 485-513, Edited by Federico M. Mazzolani, 2000

[14] Della Corte G., De Matteis G., Landolfo R., Mazzolani F. M., Seismic analysis of MR steel frames based on refined hysteretic models of connections, Journal of Constructional Steel Research, 58:1331–1345, 2002

[15] Colajanni, P., La Mendola, L., Latour, M., Monaco, A., Rizzano, G. FEM analysis of push-out test response of Hybrid Steel Trussed Concrete Beams (HSTCBs) (2015) Journal of Constructional Steel Research, 111, pp. 88-102.

[16] Latour, M., Rizzano, G. A theoretical model for predicting the rotational capacity of steel base joints (2013) Journal of Constructional Steel Research, 91, pp. 89-99.

[17] D'Aniello M., Landolfo R., Piluso V., Rizzano G., Ultimate behavior of steel beams under non-uniform bending, Journal of Constructional Steel Research, 78:144–158, 2012

[18] Ma F., Zang H., Bockstedte A., Foliente G. C., Paevere P., Parameter Analysis of the Differential Model of Hysteresis, Journal of Applied Mechanics, 71:342-349, 2004

[19] Deb K., Pratap A., Agarwal S. and Meyarivan T., A Fast and Elitist Multiobjective Genetic Algorithm: NSGA-II, IEEE Transactions on Evolutionary Computation, 6(2):182-197, 2002

[20] MultiCal: Multi-objective calibration of hysteretic models, available at www.multical.unisa.it, 2017